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SEISMIC REVIEW OF THE R. E. GINNA NUCLEAR POWER PLANT AS PART OF THE SYSTEMATIC EVALUATION PROGRAM FOR THE NUCLEAR REGULATORY COMMISSION

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ABSTRACT

This paper is a progress report on work at the Lawrence Livermore National Laboratory (LLNL) to perform a limited seismic reassessment of the Robert E. Ginna Nuclear Power Plant. The reassessment is being done for the Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program. The reassessment focuses generally on the reactor coolant pressure boundary and on those systems and components necessary to shut down the reactor safely and to maintain it in a safe shutdown condition following a postulated earthquake characterized by a peak horizontal ground acceleration of 0.2 g. Methods and modeling procedures used to analyze a complex of interconnected buildings are highlighted. However, results, conclusions, and recommendations about the ability of the structures to withstand the postulated earthquake are not presented. Such judgments will be part of the final report on the LLNL reassessment of Ginna for the NRC.

KEYWORDS: Dynamics, Earthquakes, Frames, Power Plants, Seismic Analysis, Structural Engineering

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INTRODUCTION

This paper describes work at the Lawrence Livermore National Laboratory (LLNL) to reassess the seismic design of the Robert E. Ginna Nuclear Power Plant. The reassessment includes a review of the original seismic design of structures, equipment, and components, and seismic analysis of selected items using current modeling and analysis methods, which are highlighted in this paper.

The LLNL work is being performed for the U.S. Nuclear Regulatory Commission (NRC) as part of the Systematic Evaluation Program (SEP). The purpose of the SEP is to develop a current documented basis for the safety of 11 older operating nuclear reactors, including the Ginna plant. The primary objective of the SEP seismic review program is to make an overall seismic safety assessment of the plants and, where necessary, recommend backfitting in accordance with the Code of Federal Regulations (10 CFR 50.109). The important SEP review concept is to determine whether or not a given plant

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meets the "intent" of current licensing criteria as defined by the Standard Review Plan--not to the letter, but, rather, to the general level of safety that these criteria dictate. Additional background information about the SEP can be found in Refs. 1 and 2.

Results, conclusions, and recommendations about the seismic resistance of the structures, equipment, and piping selected for reanalysis are not reported here. Such findings will be part of the final report on the LLNL reassessment of Ginna for the NRC.[3] The NRC staff will then prepare a Safety Assessment based partly on that final report.

PLANT DESCRIPTION

Owned and operated by the Rochester Gas and Electric Corporation (RG&E), the Robert E. Ginna Nuclear Power Plant is located on the south shore of Lake Ontario, 16 mi east of Rochester, N. Y. The plant is a 420-MW_e PWR that has two closed reactor coolant loops connected in parallel to the reactor vessel. The reactor containment building is a vertical, cylindrical reinforced concrete structure. It has prestressed tendons in the cylindrical wall (vertical direction only), a reinforced concrete ring anchored to bedrock, and a reinforced hemispherical dome, all designed to withstand the pressure of a loss-of-coolant accident.

A complex of interconnected buildings surrounds the containment building (Fig. 1). Though contiguous, these buildings are structurally independent of the containment building. Some of the buildings in the complex were originally categorized as Class I (vital for safe shutdown) for the purpose of the seismic design analysis, while others were considered Class III (unrelated to reactor operation or containment). Note that these classifications differ from those in Regulatory Guide 1.29,[4] which was issued after the design of Ginna. Note also that several Class I structures are connected to Class III structures. The auxiliary building (Class I) is contiguous with the service building (Class III) on the west side. The intermediate building (Class I) adjoins the service building (Class III) to the west, the turbine building (Class III) to the north, and the auxiliary building to the south. The turbine building adjoins the diesel generator annex (Class I) to the north and the control building (Class I) to the south. The facade--a cosmetic rectangular structure that encloses the dome-shaped containment structure--has all four sides partly or totally in common with the auxiliary, turbine, and intermediate buildings.

ORIGINAL DESIGN

Ginna was designed for an operating basis earthquake (OBE) characterized by a peak horizontal ground acceleration (A_{max}) of 0.08 g and for a safe shutdown earthquake (SSE) with an A_{max} of 0.2 g. Peak horizontal and vertical accelerations were assumed to be the same. Response spectra were those developed by Housner.[5] Those Class I structures and equipment that were seismically qualified were analyzed by the equivalent-static method. The maximum response acceleration of a structure or equipment item was read from the response spectrum for selected values of damping and a fundamental natural frequency. From the mass of the structure or equipment and the maximum response acceleration, the equivalent static force was obtained. This force, which represented the total dynamic effect, was then distributed along the system according to a selected shape or the mass distribution. The static response to this equivalent static force was taken to be the seismic response of the system. Responses to horizontal and vertical ground accelerations were calculated separately, then combined by direct addition. All Class I components, systems, and structures were reportedly designed to meet the stress criteria accepted as good practice and, where applicable, set forth in the ASME, USAS, ACI and AISC design standards that were appropriate then.

REASSESSMENT

The LLNL reassessment has focused on:

- The integrity of the reactor coolant pressure boundary, that is, components that contain coolant for the core and piping or any component not isolatable (usually by a double valve) from the core.
- The capability of certain essential systems and components to shut down the reactor safely and to maintain it in a safe shutdown condition during and after a postulated seismic disturbance.

The assessment of this subgroup of equipment can be used to infer the capability of such other safety-related systems as the Emergency Core Cooling System.

To review these systems, an evaluation is underway of the reactor containment building, its internal structures, and the complex of interconnected buildings (auxiliary, intermediate, turbine, control, service, and diesel generator buildings) to demonstrate structural adequacy and to obtain seismic input to equipment. Only the building complex analysis is presented in this paper.

A peak horizontal ground acceleration of 0.2 g is being used in the review analysis along with a Regulatory Guide 1.60 (Ref. 6) response spectrum for 10% of critical damping (suggested in Ref. 1). Although a probabilistic evaluation of the seismicity of the Ginna site as part of the NRC's Site Specific Spectra Study may justify a lower value, we consider a level higher than 0.2 g to be unlikely.

Table 1 lists the damping values used for Ginna together with those from R.G. 1.61 (Ref. 7) for the SSE and those recommended in NUREG/CR-0098 (Ref. 1) for structures at or just below the yield point. In general, the damping values used in the design of Ginna are lower than those now in use. One reason is that the design damping values were used to calculate OBE-based design loads, which were scaled up for the SSE evaluation. Because higher response and, consequently, increased damping are expected for the SSE, a significant degree of conservatism was typically introduced.

TABLE 1. Original and currently recommended percent of critical damping.

Structure or component	Percent of critical damping		
	Ginna (Original)	R.G. 1.61 (SSE)	NUREG/CR-0098 (Yield levels)
Prestressed concrete	2	5	5 to 7
Reinforced concrete	5	7	7 to 10
Steel frame	1 or 2.5	4 or 7	10 to 15
Welded assemblies	1	4	5 to 7
Bolted and riveted assemblies	2.5	7	10 to 15
Vital piping	0.5	2 or 3	2 to 3

REANALYSIS OF THE AUXILIARY, INTERMEDIATE, TURBINE, CONTROL, SERVICE AND DIESEL GENERATOR BUILDINGS

Current analytical techniques and computer models allow greater sophistication and treatment of detail than did methods that were available when Ginna was designed. A complete dynamic analysis of complicated structural systems such as the interconnected building complex can now be done conveniently and inexpensively. Both Class I and Class III buildings were included in the analysis because they are interconnected.

The building complex is mainly a steel frame structural system supported by concrete foundations or concrete basement structures. A typical steel frame is composed of vertical continuous steel columns with horizontal beams and cross bracing. The connections are typically bolted. The braced frames serve as the major lateral load resisting system. Several such steel frames connect various parts of different buildings, making the building complex a complicated three-dimensional structural system. The compositions of the different buildings and their interrelationships within the building complex are described in more detail in Ref. 3.

MATHEMATICAL MODEL

As described above, the braced frames comprise the principal lateral force-resisting system of the building complex. The steel framing of all the buildings are interconnected and act as a three-dimensional structural system that requires a single three-dimensional model to simulate the proper interaction effects between buildings. The model was developed based on the following assumptions:

- All buildings except the control building are founded on rock and are assumed to have rigid foundations; thus, no soil-structure interaction effects need to be considered. The control building foundation, a concrete mat supported by soil, is modeled by six linear elastic springs.

- There is no coupling betwen horizontal and vertical responses (i.e., only horizontal responses result from horizontal loadings and only vertical responses from vertical loadings).

- For the dynamic analysis, the mathematical model is designed to have only horizontal responses because the major concern is the capacity of the lateral force-resisting system. Vertical components are calculated using equivalent static loads.

- All floors and roofs are assumed to be rigid in-plane because of the high stiffness of the in-plane steel girders and concrete slabs. Each floor or roof has three degrees of freedom--two in horizontal translation and one in vertical (torsional) rotation. All points on a floor or roof move as a rigid body.

- All structural and equipment masses are assumed to be lumped at the floor or roof elevations, then transformed to the centers of gravity of each rigid floor or roof.

- Most bolted joints that connect bracing and beams to columns (and columns to base supports) are assumed to be pin or hinge connections.

- Cross bracing members, which are the primary elements of the lateral load-resisting system, are expected to buckle during compression cycles because of their large slenderness ratios. Such nonlinear behavior was accounted for in two models. In one model it is assumed that both cross-bracing members have only half the actual member cross-sectional area and can take both compression and tension during earthquake excitation. The second model was based on the assumption that bracing with the full cross-sectional area are effective in both compression and tension.

- Structures in the basement of the auxiliary building and the control building, which have concrete walls and roofs that are much stiffer than the rest of the structures, were modeled as equivalent beams.

- Only the stiffness effects of the one-story diesel generator building and the mass effects of the relatively flexible steel-frame service building were modeled.

- A uniform damping ratio of 10% of critical was assumed for the whole structural system based on the suggestion in NUREG/CR-0098 (Ref. 1) for bolt-connected steel structures under SSE loading.

In addition, the basic assumptions and model properties for the auxiliary and control buildings were adopted from a separate 1979 analysis by Gilbert and Associates, Inc. The basic assumptions listed above are described in greater detail in the final report, as are additional assumptions about individual structures.

The three-dimensional mathematical model for the building complex was prepared for the computer program SAP4 (Ref. 8). All steel frames are modeled by beam elements. The model's rigid diaphragms for all roofs and floors are represented by the rigid restraint (also called the master-slave restraint) option of SAP4. In this representation the stiffnesses of all structural members connected to the floor or roof are mathematically transformed to a master node, which we selected to be the floor or roof center of gravity.

Such a stiffness transformation, which requires no additional members or computational effort, is mathematically equivalent to the more common approach of placing infinitely rigid beams between the master node and the corresponding slave nodes of the structural members. There are 17 such rigid diaphragms in the model that were treated this way. Use of the master-slave option together with the rigid-floor assumption significantly reduces the number of degrees of freedom in the mathematical model without sacrificing its completeness.

The control building and the two-story concrete substructure of the auxiliary building are modeled by equivalent beams. The four shear walls of the diesel generator building are represented by four elastic springs attached to the north frame of the turbine building at the diesel generator building roof. The masses of the service building roof are lumped to the turbine and intermediate buildings. All other masses are lumped to the centers of gravity of floors or roofs.

The complete model has 686 nodal points, 44 dynamic degrees of freedom, 1213 beam elements, and 10 elastic springs. A three-dimensional representation of the mathematical model generated with the hidden line removal feature of SAP4 is shown in Fig. 2. Figure 3 shows the actual model representation. Further details of the model can be found in the final report.

METHOD OF ANALYSIS

The total global stiffness of the structural system was obtained by assembling the stiffnesses of all members. The total stiffness matrix has 1624 static degrees of freedom. The lumped-mass matrix was similarly obtained; however, only 44 degrees of freedom had nonzero mass.

The frequencies and mode shapes of the structural system were obtained by the subspace iteration method provided in SAP4. Since there are only 44 nonzero-mass degrees of freedom, the structural system has only 44 independent modes. By requesting solutions for all 44 modes, the subspace iteration method reduces to the standard Guyan reduction, and the iteration process converges in the first step. The frequencies and mode shapes can be extracted inexpensively. The frequencies and the ten largest modal participation factors are listed in Table 2. Representative mode shapes are shown in Figs. 4 and 5.

After the frequencies and mode shapes were obtained, the structural responses were computed by the response spectrum method. The seismic input was defined by the horizontal spectral curve of the SSE specified in R.G. 1.60 for 10% structural damping and 0.2 g peak acceleration.

Two structural models were analyzed, one with half the bracing area (half-area model), one with the full bracing area (full-area model). For each model, two analyses were performed, one with the input excitation in the N-S direction and the other in the E-W direction. In each analysis, 44 structural modes were included, and for each direction the modal responses were combined by the square-root-of-the-sum-of-the-squares (SRSS) method. Responses due to N-S and E-W excitations were also combined by the SRSS method. Vertical responses were obtained by taking 13.3% (0.2 g x 2/3) of the dead load responses.

IN-STRUCTURE RESPONSE SPECTRA

A direct method was applied to generate seismic input spectra for equipment at various locations in the structure.[9,10] This method treats earthquake input motions and the response motions as random processes. The

TABLE 2. Modal frequencies of the interconnected building model.
Note: Numbers in parantheses are the ten largest modal participation factors in the E-W and N-S directions, respectively.

Mode No.	Frequency, Hz: Half-area model	Frequency, Hz: Full-area model
1	1.8 (3.4, 12.9)	2.3 (7.4, 12.6)
2	2.0 (10.2, 0.2)	2.4 (8.5, 4.7)
3	2.1	2.8
4	2.4	3.1
5	2.6	3.2 (7.4, 0.6)
6	2.8	3.4
7	2.9	3.4
8	3.3	3.6
9	3.4	3.9
10	3.6	4.0 (6.3, 1.4)
11	4.0	4.3
12	4.2	4.3
13	4.2	4.6
14	4.4	4.6
15	4.7	5.4
16	5.6	6.7
17	6.1	6.9 (12.7, 6.4)
18	6.5 (6.4, 4.5)	7.0
19	6.6	7.3
20	6.7 (8.4, 8.5)	7.4
21	6.9 (10.3, 7.2)	7.5
22	7.0	8.0
23	7.8	9.7 (5.1, 8.3)
24	9.3	10.4
25	9.5 (5.4, 8.4)	10.6
26	10.4	10.9
27	10.8	11.1
28	11.1	11.7
29	11.2	12.1
30	12.2	12.8
31	13.5	14.0
32	13.8	16.4
33	16.4	16.7
34	17.8 (2.4, 6.6)	17.8 (2.3, 6.5)
35	18.5	18.6
36	19.3	19.5
37	21.1 (0.1, 27.1)	21.2 (0.1, 27.1)
38	22.9 (26.9, 0.1)	22.9 (26.9, 0.1)
39	27.0	27.2
40	33.5	33.6
41	41.2	41.2
42	45.1	45.7
43	57.8	57.8
44	60.4 (6.7, 0.0)	60.4 (6.7, 0.0)

response spectral curve at any location in the structure can be derived from the frequency response function of an oscillator, the frequency response function of the structure at that location, and the input ground response spectral curve. This method avoids the troublesome task in the time-history approach of selecting proper corresponding time-history input for the specified response spectrum. Typical in-structure spectra are shown in Figs. 6 and 7.

GENERAL RESULTS

In comparing the frequencies and modal participation factors for all 44 modes of the full-area and half-area models, we found, as expected, that modes with low frequencies are those dominated by steel parts of the structural system (i.e., the framing system) and that high-frequency modes are dominated by the concrete structures (i.e., the control building and the basement structures of the auxiliary building). Also, as expected, the difference between the half- and full-area models is apparent only at low frequencies or for steel structure modes. High-frequency modes are almost identical for both models.

Several high-frequency modes have significant modal particpation factors. In fact, the modes having the highest factors in the N-S and E-W directions are the 37th and 38th modes, respectively (See Table 2). Inclusion of the high-frequency modes is therefore necessary, especially in computing the in-structure response spectra.

Comparisons of member forces between the two models show that bracing forces are generally lower in the half-area model than those in the full-area model, but the reverse is true for column forces.

SUMMARY

This paper highlights the model and methods used to perform a dynamic seismic analysis of an interconnected building complex at the Robert E. Ginna Nuclear Power Plant. The analysis was done as part of the NRC's Systematic Evaluation Program. The detailed three-dimensional beam-element type of model and the underlying assumptions made to develop it are presented. The assumption of in-plane rigidity for 17 floor and roof diaphrams along with use of the master-slave option of the SAP4 computer code for calculating modal properties greatly simplified the computational effort. Moreover, the model allows direct computation of member forces, thus eliminating additional steps necessary when an equivalent stick type of model is used. Representative mode shapes and floor-response spectra are shown, and general results are reported. However, detailed conclusions about the seismic resistance of the building complex are to be found in the final report to the NRC on this work (Ref. 3).

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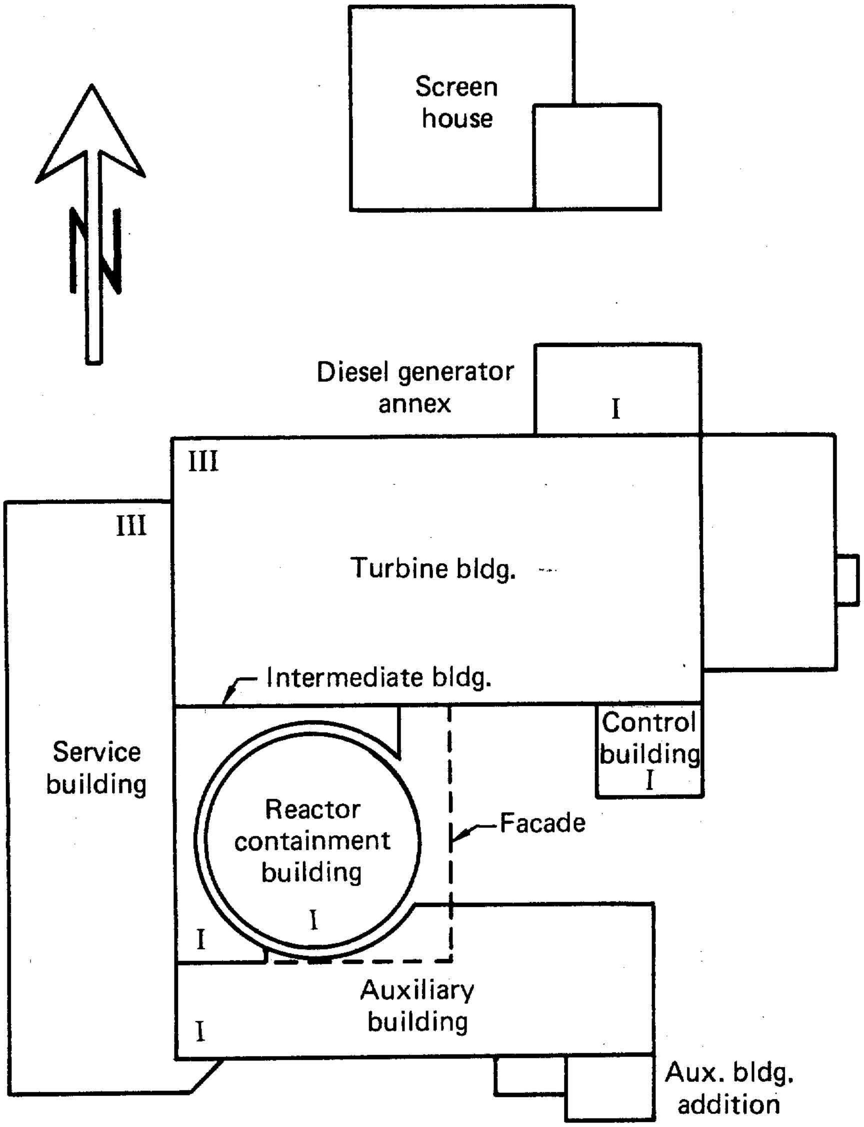

FIG. 1. Schematic plan view of the major Ginna structures shows the structurally independent containment building and the complex of interconnected seismic Class I and Class III structures.

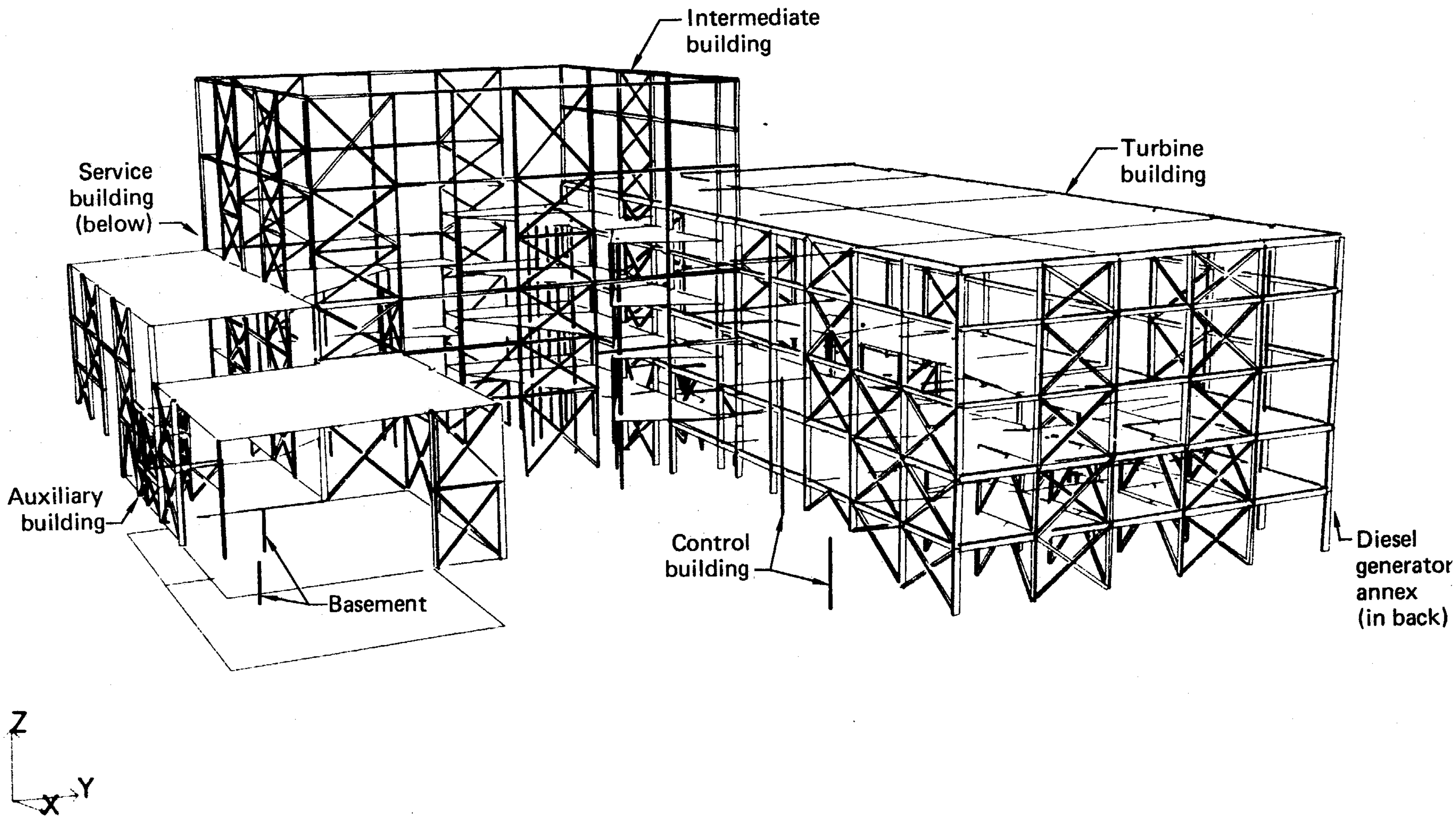

FIG. 2. Three-dimensional representation of the interconnected building complex (generated by the hidden line removal feature of SAP4) shows floor and roof levels.

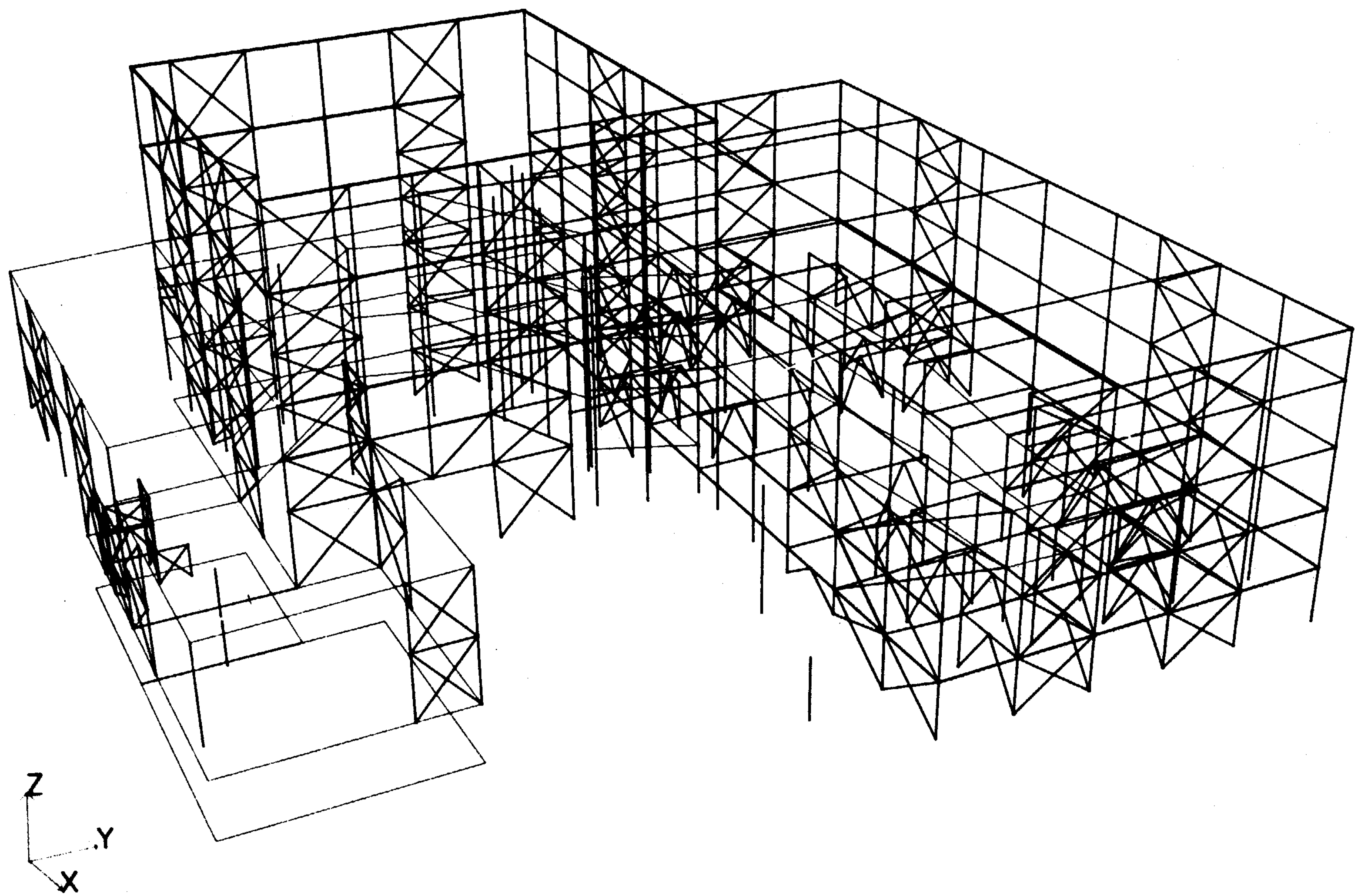

FIG. 3. Details of the framing are shown in the three-dimensional SAP4 model of the interconnected building complex, which has 1213 beam elements and 686 nodes.

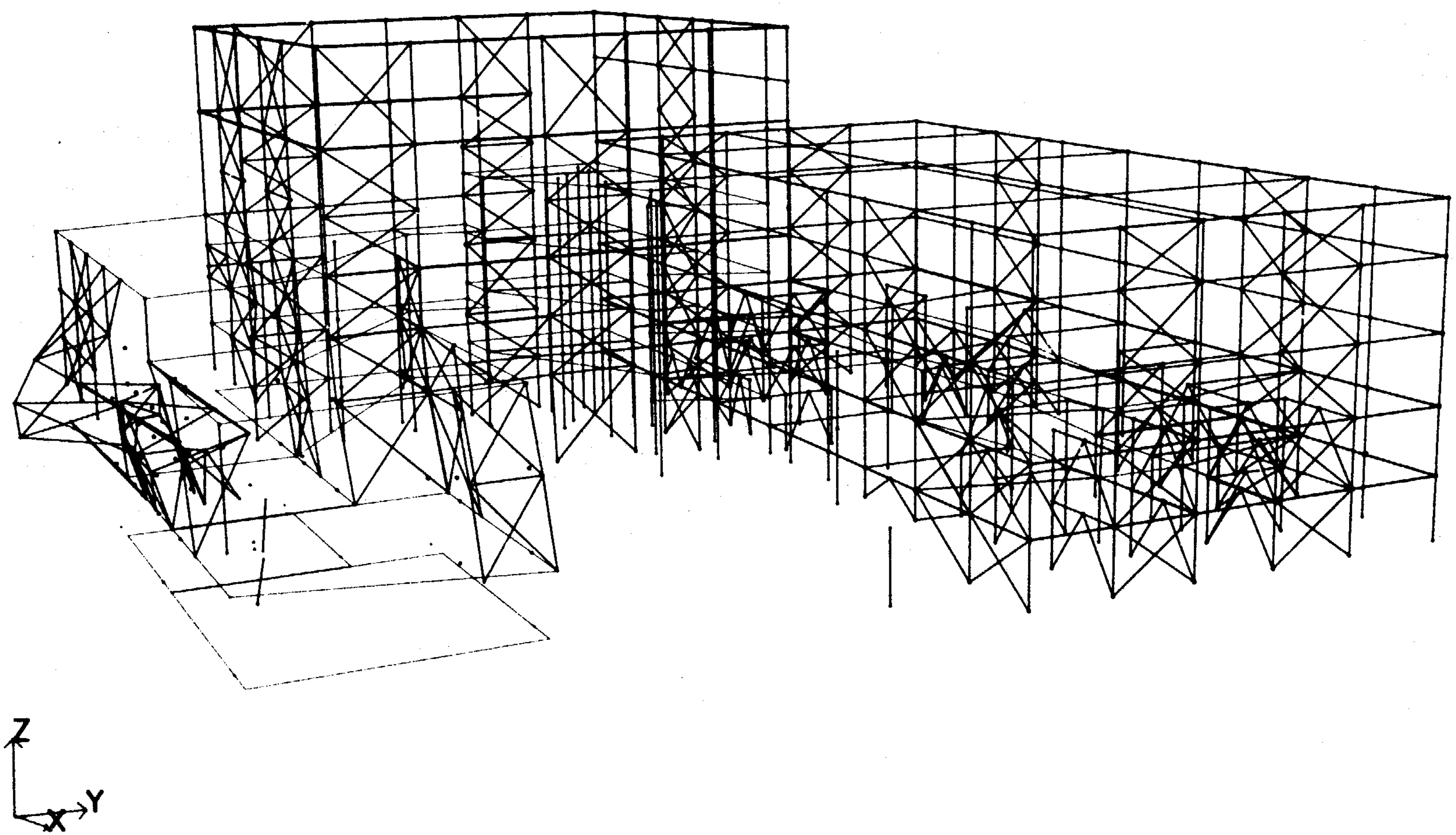

FIG. 4. The shape of Mode No. 38 (full-area model, frequency = 22.9 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

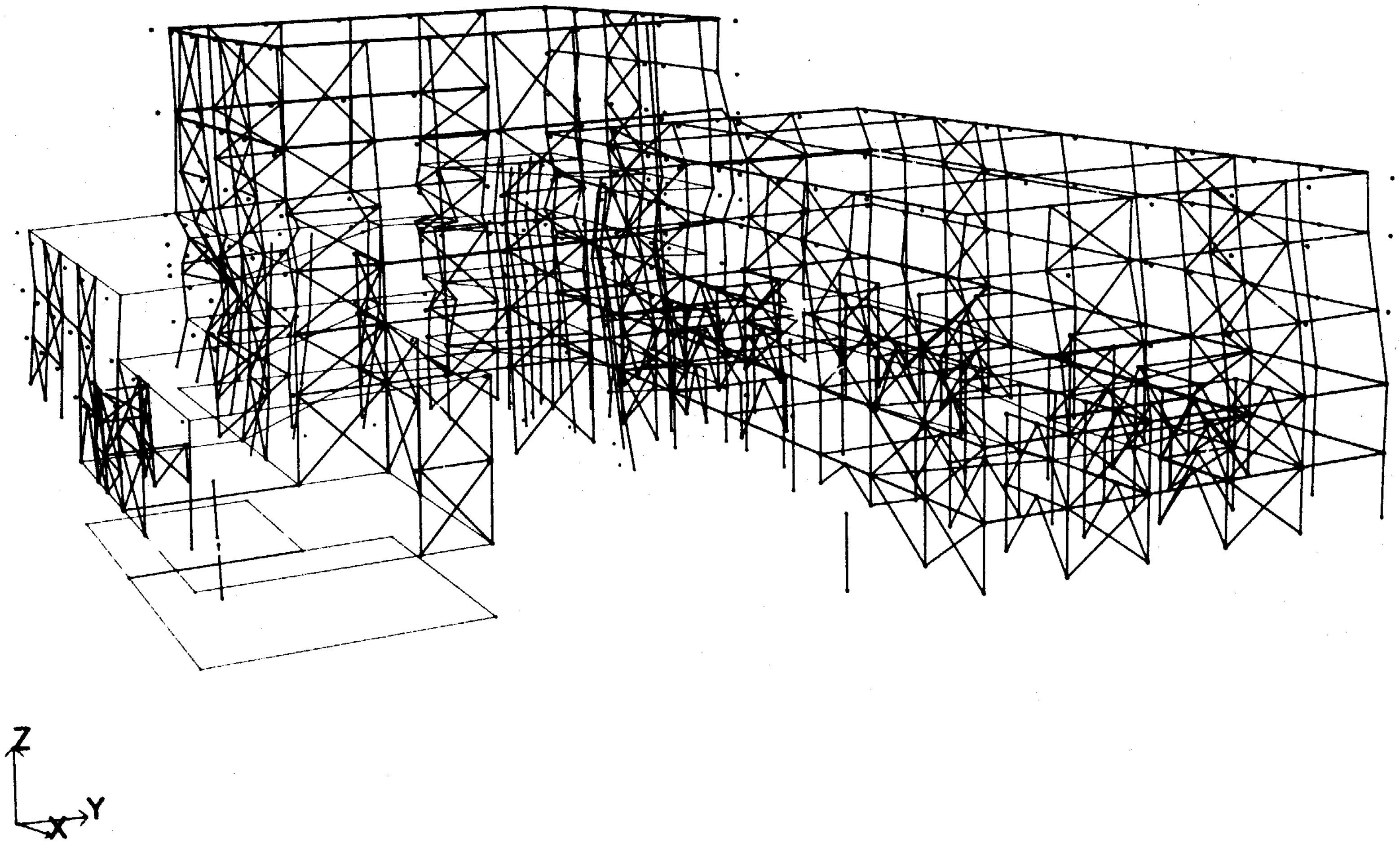

FIG. 5. The shape of Mode No. 2 (full-area model, frequency = 2.4 Hz). Note that distortion from the original (dotted shape) is exaggerated for clarity.

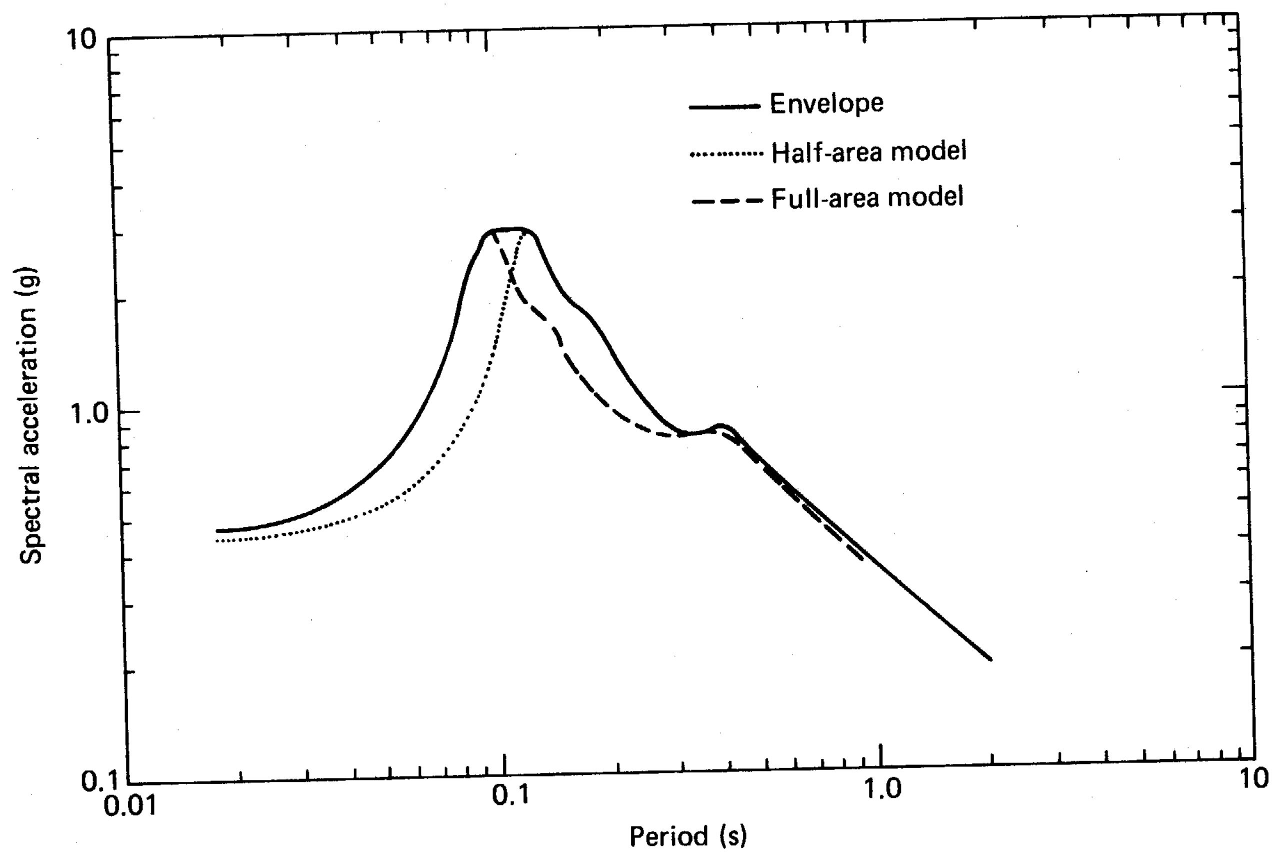

FIG. 6. E-W in-structure response spectra at 3% of critical damping for the auxiliary building platform were generated by a direct method for both full- and half-area models.

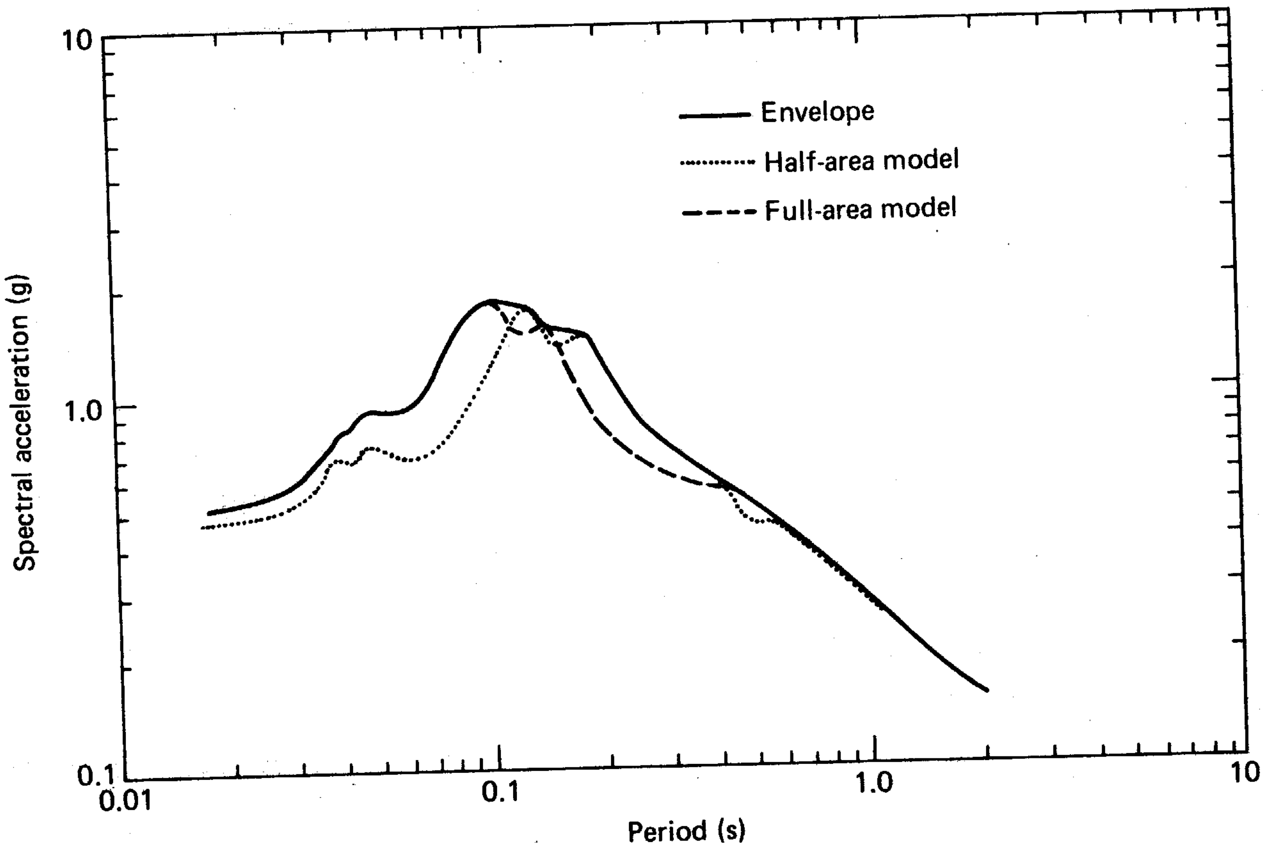

FIG. 7. N-S in-structure response spectra at 7% of critical damping for heat exchanger 35 (located on the platform) were generated by a direct method for both full- and half-area models.